HHS Public Access
Author manuscript
Int J Min Sci Technol. Author manuscript; available in PMC 2016 October 06.

Published in final edited form as:
Int J Min Sci Technol. 2016 January ; 26(1): 23–30. doi:10.1016/j.ijmst.2015.11.005.

Analysis of alternatives for using cable bolts as primary support at two low-seam coal mines

Gabriel S. Esterhuizen* and **Ihsan B. Tulu**
Office of Mine Safety and Health Research, NIOSH, Pittsburgh, PA 15236, USA

Abstract

Cable bolts are sometimes used in low-seam coal mines to provide support in difficult ground conditions. This paper describes cable bolting solutions at two low-seam coal mines in similar ground conditions. Both mines used support systems incorporating cable bolts as part of the primary support system. Two original cable bolt based support systems as well as two modified systems are evaluated to estimate their ability to prevent large roof falls. One of the support systems incorporated passive cable bolts, while the other used pre-tensioned cable bolts. The results and experience at the mines showed that the modified systems provided improved stability over the original support systems. The presence of the cable bolts is the most important contribution to stability against large roof falls, rather than the details of the support pattern. It was also found that a heavy steel channel can improve the safety of the system because of the ‘sling’ action it provides. Additionally, the analysis showed that fully-grouted rebar bolts load much earlier than the cable bolts, and pre-tensioning of the cable bolts can result in a more uniform distribution of loading in the roof.

Keywords

Roof support; Coal mining; Cable bolt; Numerical modeling

1. Introduction

Cable bolting is sometimes used as primary support in coal mines experiencing difficult roof conditions. In low-seam mines the flexibility of the cable bolts allows greater length supports to be installed near the advancing face without the use of couplers. When used as primary support, the cables are typically installed in the same row as fully grouted bolts, replacing two or more of the bolts in each support row. A heavy steel channel may be used as a strap to spread the support load over a greater portion of the roof. Historically, the Mine Safety and Health Administration (MSHA) has not allowed widespread use of partially grouted un-tensioned bolts (e.g., passive cable bolts) for primary support; however, pre-tensioned cable bolts have been accepted.

Various solutions using cable bolts as primary support were attempted at two low-seam coal mines in Western Pennsylvania that were experiencing difficult roof conditions. Both mines

*Corresponding author. Tel.: +1 412 3865207. eee5@cdc.gov (G.S. Esterhuizen).

originally used fully grouted rebar bolts as primary support and cable bolts as supplementary support. It was found that when a large roof fall occurred, the cable bolts may be contained within the dome of fallen rock. As problematic roof conditions continued to exist, both mines decided to use cable bolts as part of the primary support system. The cable bolts were located near the ribs of the entry, to increase the likelihood that they would be anchored outside the dome of potentially unstable roof. The cable and rebar bolts were installed on a heavy steel channel that acts as a "sling" to distribute the load across the width of the entry. At the first mine, Mine A, pre-tensioned cable bolts were used while at Mine B, un-tensioned cables were used. The two support systems consist of essentially the same support components installed in different patterns and with varying degrees of pre-tension.

The original and modified support systems were selected for analysis as part of current research into roof support design at the NIOSH Office of Mine Safety and Health Research (OMSHR). The objective of the analyses was to determine whether there was a significant difference in the potential of the support systems to prevent large roof falls. The analyses were focused on large roof falls in which the height of roof collapse extends more than 90 cm above the roof line of the entry, and typically extends above the bolted horizon. Smaller roof falls that occur between bolts or that are associated with individual geological structures are excluded from the analyses.

The effectiveness of the support systems was initially evaluated using an equation that estimates the stability factor an entry against large roof falls [1]. The initial assessment was followed up by FLAC3D numerical model analyses to investigate the contribution of the different support units to roof stability. The models also provided insight into the likely modes of roof and support failure. Scenarios without any support, using fully grouted bolts only, and cable bolts with fully grouted bolts were considered.

2. Geotechnical parameters

The two case study mines both extract the Lower Kittanning coalbed. The mines use the room-and-pillar method in a mining height of about 1.2 m. The depth of cover is approximately 120–150 m at both mines. In certain locations of the two mines the roof consists of laminated, dark gray, silty shale that is associated with difficult ground conditions.

3. Geology

The silty shale responsible for difficult ground conditions can be up to 10 m thick and may contain sandstone intrusions. It is overlain by a stronger interbedded sandstone and shale unit. Sandstone is occasionally found close to the coalbed being mined, but typically was found no closer than 2.4 m above the coal bed in the area studied at Mine B. Observations of the rock exposed in roof falls show that it tends to delaminate in thin slabs that are about 25–75 mm thick. Fig. 1 shows the delaminated roof exposed at Mine A and Fig. 2 shows the laminated shale exposed in a roof fall at Mine B.

Point load strength testing of the silty shale roof at Mine A showed that the compressive strength is approximately 55–60 MPa perpendicular to the bedding. Uniaxial compressive strength tests at Mine B showed higher strength, but underground observations and index testing of roof rocks indicated that the lower strength determined at Mine A was likely to be more representative of the observed roof response. These properties are similar to the Lower Kittanning roof properties published by Zhang et al. [2].

The available rock strength and bedding information were used to classify the rock mass using the coal mine roof rating (CMRR) [3]. The CMRR classification of the silty shale roof is as follows: uniaxial compressive strength (UCS) of roof rocks = 55–60 MPa, rating = 17; bedding strength rating (weak planar), rating = 16; bedding intensity rating (bedding spacing 25–75 mm), rating = 12; and total unadjusted CMRR unit rating, unit rating = 45.

Owing to variability of the rock strength properties, the unit rating can be expected to vary between about 40 and 50. For the purpose of the analysis, the average values shown above were used.

4. Horizontal stress

Stress measured in the vicinity of the two mines shows results typical of Northern Appalachia with a relatively high pre-mining horizontal stress associated with regional tectonic loading [4,5]. At Mine A, the major horizontal stress is estimated to be oriented N70E and at the Mine B it is estimated at N80E. The orientation of the major horizontal stress is considered in the mine layouts. Where possible, the mining direction is oriented so that the development is directed favorably relative to the stress field. For the analysis of the support systems, it was assumed that the entries were developed in a horizontal stress field associated with tectonic strain components of 0.0005 and 0.0006.

5. Large roof falls

At Mine A, forty large reportable roof falls occurred over a period of ten years. The falls occurred in spite of intensive roof support in the form of primary rebar bolts and secondary cable bolts. Primary bolts up to 2.1 m long and cable bolts up to 4.8 m in length were used. Despite these efforts, falls were typically in the north–south orientation and would progress upwards to the top of the laminated shale, which was typically about 3.6 m above the mine roof. Fig. 2 shows the laminated nature of the collapsed roof at Mine A. The presence of cutter roof and other signs of stress-induced roof damage confirm that the mine was located in a relatively high horizontal stress field, often encountered in mines in the Northern Appalachian area.

At Mine B, roof control in the areas where the low-strength silty shale is present was generally satisfactory until a 300-m-long roof fall occurred in 2013. The roof also exhibits signs of excessive horizontal stress, with cutters and stress fractures observed. Fig. 3 illustrates the roof damage caused by a cutter that formed well outby the advancing faces at Mine B. The mine layout is adjusted so that the development direction is favorable relative to the major horizontal stress. Prior to the major roof fall, cable bolts were used as part of the primary support system. The large roof fall prompted a change in the mining layout and support system.

6. Support systems evaluated

Several different roof support systems had been employed at the two mines. The original support systems generally consisted of various combinations of conventional primary bolts and cable bolts as supplementary support. The modified support systems introduced cable bolts as part of the primary support installation. In order to gain an understanding of the general ground response at these mines and to gauge the effects of the various supports implemented at these mines, a series of six scenarios were modeled:

System 0: unsupported entry: initially the stability of an entry without any support was assessed. The entry width was 6 m. The unsupported entry is labeled “UNSUP-W20” in the plotted results.

System 1: entry supported by fully grouted bolts only: the effect of using four 1.8-m-long fully grouted conventional bolts as primary support was assessed. The bolts were assumed to be untensioned. The results provide a base case so that the benefit of the cable bolts can be determined. This support system is labeled “4B W20” in the plotted results.

Support systems 2 and 3 are the original support systems used at the case study mines, in which alternating rows of cable bolts were used. A schematic illustration of these support systems is presented in Fig. 4.

System 2: fully grouted conventional bolts and un-tensioned cable bolts as supplementary support: the original support system selected for analysis at Mine A consisted of fully-grouted 2.0-m-lomg, conventional bolts with ultimate tensile strength of 170 kN. These bolts are installed through a T3 channel. The bolts are located 1.2 m apart in rows that are 1.2 m apart. In addition, there are two cable bolts spaced 2.4 m apart in rows that are 1.8 m apart. The cable bolts are 3.6 m long, 17 mm-diameter with 270 kN ultimate tensile capacity. The cable bolts are anchored using 1.2 m of resin grout and are un-tensioned. Entries are 6 m wide. This system is labeled “4B2C-W20” in the plotted results.

System 3: fully grouted conventional bolts and tensioned cable bolts as part of primary support with alternating locations: at Mine B the initial support system in the area of concern consisted of alternating rows of conventional bolts, cable bolts, and T3-channels installed on-cycle. Two patterns of bolts and straps were installed. The first row of the pattern consisted of two, 1.8-m-long, No. 5 tensioned rebar bolts through the center section of the entry and two, 3.6-m-long, 17-mm-diameter cable bolts on the outside. The second row reverses the order. Bolt tension is approximately 50 kN. There are 4 support units in a row and the rows are 1.2 m apart. Entries are 5.5 m wide. This system is labeled “4B2C-W18” in the plotted results.

Support systems 4 and 5 are the final support systems which make use of bolts and cable bolts in the same row as primary supports at Mine A and Mine B respectively.

System 4: fully grouted conventional bolts and tensioned cable bolts as part of primary support: the modified support system at Mine A consisted of two, 2.0-m-long bolts and two, 3.6-m-long cable bolts installed on-cycle on a T3 channel. Support rows are 1.2 m apart. The cables are located outside of the conventional bolts, near the pillar ribs. The cables bolts are anchored using 1.2 m of resin and are tensioned to 50 kN. Entries are 4.9 m wide. This system is labeled “2B2C-W16” in the plotted results.

System 5: fully grouted conventional bolts and un-tensioned cable bolts as part of primary support: the modified support system at Mine B consisted of support rows with four 1.8-m-long No. 5 tensioned rebar bolts and two 3.6-m-longpassive cables installed on a 4.3-m-long T3 channel. The cables are located about 50–60 cm from the ribs, near the extremities of the channel. Additionally, the entry width was reduced to 4.9 m. This system is labeled “4B2C-W16” in the plotted results.

7. Initial assessment of support systems

A recently developed equation for predicting the stability factor of the roof of coal mine entries against large roof falls was used to conduct an initial assessment of the various support systems [1]. The equation predicts the Stability Factor (SF) that would be obtained by numerical models using a simple spreadsheet type calculation. The SF values are based on the results of numerical model analyses using the Strength Reduction Method (SRM) as adopted for coal mine entries [6]. Using this approach, the SF values of support systems currently used in coal mines with poor to moderate roof conditions typically fall in the range of 1.6–2.4 using the SRM-calculated SF values.

The numerical models used to develop the prediction equation simulated entries supported by fully-grouted, solid bar bolts and partially-grouted cable bolts. The prediction equation can therefore also be used to assess these types of supports. The equation does not discern between tensioned and un-tensioned supports, nor does it account for the exact location of support units in the roof. However, it does consider the rock mass strength, depth of cover, horizontal stress conditions, entry width, length of support, type of support, and density (number) of supports installed. It is therefore suitable for rapidly assessing the SF associated with the various support systems used at the two case study mines.

8. Assessment results

The assessment of the support systems was conducted for a depth of cover of 150 m. Because of the uncertainty about the actual value of the horizontal stress at the mines, two different horizontal stress scenarios were evaluated. In the discussion below, the averages of the SF values calculated for the two stress scenarios are used. Fig. 5 presents the assessed SF values for the five support systems calculated using the prediction equation. It can be seen from the results that the range of horizontal stress conditions considered can produce a variation in SF of about 0.5. The results for the individual cases are summarized below.

a. *System 0 (UNSUP-W20):* the roof of the unsupported entry has an SF of 1.46, which falls below the typical range of SF values for supported entries. This confirms that support is required to prevent large-scale roof falls.

b. *System 1 (4B W20):* the results for the system with four 1.8-m-long conventional bolts suggest that the addition of bolts alone does little to improve the SF of the roof against large-scale collapse since the SF is increased to only 1.62. This SF value is at the lower end of current roof support practice, indicating that additional support may be required.

c. *System 2 (4B2C W18):* the 4-bolt and 2-cable bolt system causes a significant improvement in the SF to 2.06. The improvement is attributed to the cable bolts which are located in rows 1.8 m apart.

d. *System 3 (2B2C W20):* this system is assessed to have an SF of 2.21. This higher value is a result of the cable bolts row spacing of 1.2 m in this system, compared to 1.8 m in System 2.

e. *System 4 (2B2C W16):* this result shows the impact of reducing the entry width from 6.0 to 4.9 m and simultaneously reducing the cable bolt spacing from 1.8 to 1.2 m. A considerable increase in SF occurs, to a value of 2.70.

f. *System 5 (4B2C W16):* this system used at Mine B mine is similar to the system at Mine A (System 4) except that four, fully grouted, conventional bolts are used and the cable bolts are not tensioned. As expected the SF is greater at 2.84. Note that the assessment equation does not consider the application or absence of pre-tension.

9. Discussion of assessment results

The assessment equation provides useful insight into the likely roof stability associated with the various support systems. The results confirm that fully-grouted conventional bolts alone are unlikely to be sufficient to support in the specified ground conditions. The positive impact of using cable bolts is clearly shown, increasing the stability factor from around 1.6 to over 2.0 for support systems 2 and 3. Reducing the spacing between rows of cable bolts produces a further improvement in the stability factor and the positive influence of reducing the entry width is also clearly demonstrated.

The assessment equation is limited in some respects, but provides useful information for comparing support alternatives based on the predicted SF values. The trends in the results are clear and agree with the practical experience at the mines. The positive impact of cable bolts is clearly shown in the results for systems 2 and 3. The further impact of reducing entry widths from around 6.0 m to 4.9 m is also clearly shown. These relative improvements in stability were experienced at the operating mines.

10. Numerical model analysis

The initial assessment of the support systems was supplemented by conducting numerical model analyses of each system. Numerical models are finding increased application in the analysis of support alternatives for coal mine entries [7,8]. The objective of the analyses was to investigate the reasons behind the changes in SF values seen in the rapid assessment results. The numerical models also allowed investigation of the load sharing that occurs between the different support types, the impact of pre-tension, the likely mode of failure of the roof, identification of the weakest link in the support system, and the effect of the different patterns of cable bolt installation.

The model analyses were conducted using the FLAC3D finite difference software by implementing the Strength Reduction Method (SRM) as modified for coal mine entry analysis [6]. The SRM allows the SF of the entry roof against large-scale collapse to be determined. Fig. 6 shows a typical model, which simulates a slice through the supported entry, with the conventional bolts, cable bolts and T3 channels located at their appropriate positions. The SRM-based SF results have been validated against field data and empirical design techniques and have been demonstrated to provide realistic estimates of expected roof stability in coal mine entries [1,9]. The results of the SRM provide outcomes associated with the rock and support loads when the roof is at a critical point of stability, just prior to collapse. This critical stability condition is particularly useful for identifying the contribution of the different support components to overall stability. It is possible to identify which support components have already failed, which supports are still carrying load, why the roof ultimately collapses, and what might be done to improve the system.

The input parameters for the numerical models are based on the UCS and the CMRR unit ratings of the roof rocks. In these models, the roof was assumed to consist of laminated silty shale, described earlier, with a unit rating of 45. Details of the method used to derive model inputs are presented in Esterhuizen et al. [9]. The main inputs for modeling the silty shale are summarized in Table 1. Two values for the rock mass cohesion are given; the first describes the cohesion when the rock is loaded perpendicular to the bedding and the second is when the rock is loaded parallel to the bedding. The lower cohesion value is used to account for the reduced strength in the roof of an excavation caused by delaminating and buckling of the individual sedimentary layers in a geologic unit.

The bolts and cable bolts were modeled using the strength parameters published by the different manufacturers, as listed in Section 6. For the cable bolts, the elastic modulus of the free length of steel was reduced by a factor of 2.0 to simulate the reduced stiffness associated with the “unwinding” of the cable strands under high loads, as seen in controlled field tests. Grout axial and lateral stiffness were modeled based on the procedures described in Tulu et al. [10]. The “grip factor” for the grout was set at 8 kN/cm, which ensured that the 1.2 m grouted length of the cable bolt exceeded its ultimate strength.

The vertical stress in the models was based on the depth of cover and the density of the strata. The horizontal stress was based on the results of stress measurements in the US coal regions, as summarized by Mark and Gadde and Dolinar [4,5]. According to these studies, the horizontal stress in a rock unit is related to the horizontal tectonic strain in the rock mass. Because of the uncertainty of the actual stress conditions, various stress scenarios were evaluated in which the horizontal tectonic strain was varied between 0.0004 and 0.0006, as indicated for mines in Northern Appalachia. Further details of the model setup, boundary conditions, and method of implementing the SRM are presented in Esterhuizen [6].

The results presented here focus on the response of the support systems when the entry is at a critical point of stability, just prior to collapse of the roof. In the models, the strength of the roof rocks is reduced to the point of failure. In practice, such a point of critical stability can be reached if the variable rock strength and variable stress field produce a situation in which the rock and support system strength are matched by the gravity and stress-driven forces acting on the roof strata. Under such a critical stability condition, a small change in either the rock strength or the imposed loads can produce a large roof fall. The model results enable examination of how the bolt loads and rock failure develop to produce such a critical condition. It is also possible to identify the critical component that is likely to fail and to produce a large roof fall.

11. Analysis of support systems 2 and 3

The performance of the fully-grouted, conventional bolt and cable bolt support systems 2 and 3 were evaluated as part of this study. Fig. 7 shows the bolt load versus roof sag for the critical stability condition just before the roof collapses for the two systems. It can be seen that system 2 reaches its critical loading state after about 3 in. of roof sag, while system 3 accommodates up to 7 in. of roof sag. This is not necessarily an advantage, since the roof yield has extended above the two inner cable bolts and they are not achieving their full load capacity.

System 2

Fig. 7a shows that one of the cable bolts has reached its maximum tensile load. The other cable bolt is not fully loaded yet. Looking at the development of the bolt loads, one can see that the conventional bolts achieve their maximum load of about 15–17 ton after about 0.5 in. of roof sag. As yielding of the roof rock continues to develop upwards, the conventional bolts start to shed load as they become encapsulated by the yielding roof rocks. The two inner conventional bolts lose most of their load at about 50 mm of roof sag. At about 50 mm of sag, the passive cable bolts have generated about 130 kN of load, similar to the conventional bolts, which are unloading at this stage. When the roof sag exceeds 50 mm, the cable bolt load increases rapidly, with one of the cable bolts achieving its maximum tensile strength and appears to have started to yield. It seems that the passive cable bolts and fully grouted, conventional primary bolts act as two individual systems, with the impact of the cable bolts dominating after the fully-grouted, conventional bolts start to shed load.

System 3

the roof sag and bolt load curves shown in Fig. 7b demonstrates that the pre-tension of the cable bolts in system 3 causes the conventional bolts and cable bolts to respond (develop loading) at approximately the same rate. Both support types achieve high loads at less than

25 mm of roof sag. However, the high loads are only achieved by the two inner cable bolts, while the two outer cable bolts remain at a relatively low stress. The two inner conventional bolts are slowly yielding, because their anchorage zone is located within yielding rock. The height of yield of roof rocks is highest near the center of the entry and envelops the two inner cable bolts sooner than the outer bolts.

Fig. 8 shows the extent of rock damage and bolt loads at the critical stability condition for support systems 2 and 3. The distribution of bolt loads can clearly be seen. The gray shading is an indicator of roof sag, showing much greater roof sag being accommodated by system 3. Bedding shear and rock fracture is also indicated in the diagrams. The two inner cable bolts of system 3 can be seen to be encapsulated within the sagging dome of yielded rock. Some floor yield is indicated in both cases.

12. Analysis of support systems 4 and 5

Support systems 4 and 5 have cable bolts near the ribs of the entries connected by the T3 channel. In system 5, the cable bolts are not tensioned while in System 6 the cable bolts are tensioned. Results are presented for the case in which the systems are at a critical stability condition, just prior to collapse of the roof.

System 4

Fig. 9 again shows that the pre-tensioned cable bolts in system 4 respond much sooner than passive cable bolts. The cable bolts and fully grouted bolts achieve significant loading at about 25 mm of roof sag. The roof sag is arrested after about 250 mm of sag. The fully-grouted bolts start to shed load when the roof sag exceeds about 25 mm. The cable bolts and fully grouted bolts initially act as a combined system. Once the roof yield has progressed above the grouted bolts, the cable bolts maintain a significant load.

System 5

the results show that roof sag of about 375 mm can be accommodated before the critical stability condition is reached. This is thought to be related to the added reinforcement provided by the four pre-tensioned and fully-grouted conventional bolts, compared to only two fully-grouted conventional bolts in system 4. Here the passive cable bolts are seen to act like a separate support system, only becoming active after the fully-grouted conventional bolts have shed most of their load.

Fig. 10 shows the rock yield and bolt loading at the point of critical stability for support systems 4 and 5. Fig. 10 clearly shows how the T3 channel acts as a sling between the two cable bolts. In both cases the fully grouted bolts provide only limited support or reinforcement to the yielding roof. The sling action can produce large tensile stresses in the T3 channel, which may shear the head of the cable bolt [11]. Fig. 3 shows an example of the absence of cable bolts along the right side of the entry, which may have sheared in this manner.

Fig. 11 shows a case where significant roof sag occurred and the damaged roof is held in position by the sling-effect of the cable bolts and the T3 channel. In Mine B, roof sag was caused by roof yield that has apparently developed above the fully grouted bolts. The roof is supported from the two outer cable bolts, and the T3 channel acts as a sling. Given the low mined height, the mine operational staff could readily identify areas which sagged. When this condition was observed, cribs or other types of standing supports were typically installed to arrest further movement. So, although the support system did not prevent roof sag it did provide a warning of damaged roof allowing remedial actions to be taken to prevent collapse.

13. Discussion

The numerical model analyses provide insight into the contribution of each support element and the overall performance of the supported roof. It is shown that the use of passive cable bolts produces a dual-mode support system. Initially the fully-grouted conventional bolts load-up while the passive cable bolts only make a small contribution to supporting the roof load. The cable bolts start to develop significant loading only when yielding of the roof progresses above the conventional bolts. In this role, the cable bolts may be seen as a "backup system" that is designed to prevent a collapse of the roof after the conventional primary bolts become ineffective. The heavy steel channel is an important component of the support system, providing a sling to hold the damaged and sagging roof rock in place.

In support system 2, where passive cable bolts are located 1.2 m on either side of the entry center line, the cable bolts develop loading relatively quickly but can become encapsulated within the dome of failed rock. Collapse of the roof is indicated when the dome of yielded rock extends above the cable bolts and the bolts simply fall out with the damaged roof.

In system 3, where cable bolts are located alternatively at the inner/outer positions, it appears that the inner cable bolts will become loaded before the outer ones. The danger is that the inner two cable bolts may become overloaded, because they are now required to carry the dead weight of the roof over a distance of 2.4 m along the entry. This may have been a contributing factor to the 300-m-long fall that occurred at Mine B.

The experience at the two mines as discussed in this report led to the solution to locate the cable bolts near the entry ribs, as in systems 4 and 5. According to the numerical model results, this moved the cable bolts away from the dome of yielding roof rocks, allowing them to provide anchorage in relatively less-damaged strata. The cable bolts at this location again serve as a backup support system, arresting the sag of the damaged roof if the fully grouted bolts lose their effectiveness. System 5 with its greater number of fully-grouted conventional bolts provides the greatest capability for accommodating roof sag.

At Mine B, it was observed that after changing from system 3 to system 5, occasional roof sagging was observed, but the sling action of the cable bolts and T3 channel successfully controlled the damaged roof, preventing another large, running roof fall from developing.

14. Conclusions

Numerical model-based methods have been used to evaluate the relative merits of four roof support systems used at two low-seam coal mines. The mines experienced difficult roof conditions and large roof falls when mining under laminated, silty shale. In spite of the low-seam working condition, cable bolts were installed without much difficulty and responded with mixed success during the early trials. The support systems at both mines were independently modified so that cable bolts are installed on-cycle as part of the primary support system. Both mines also discovered that locating the cable bolts near the entry ribs resulted in successful ground control.

An initial analysis of the support systems was conducted using a recently-developed prediction equation that estimates the SF of coal mine entries against large roof falls. The results showed that in both cases the SF of the modified support systems exceeded those of the original support systems. The increased SF values are confirmed by the increased stability experienced during mining operations with the modified systems.

Detailed numerical model analysis was able to identify the contribution of each support element to roof stability. The likely causes of roof collapse and support system failure could be identified.

It was shown that using passive cable bolts results in a dual-mode support system, where the cable bolts become active and the dominant supporting component after the fully-grouted conventional bolts have shed load. The cable bolts can be seen as a “backup system” to the primary support system.

The importance of a heavy steel channel connecting the cable bolts was clearly demonstrated. The channel acts as a sling, holding the damaged roof in place so that appropriate remedial measures can be taken.

The model results show that cable bolts located near the entry ribs are moved away from the dome of yielding rock over the entry. This cable location appears to result in improved stability compared to cable bolts that are installed within the yielding dome.

Practical experience at the operating mines confirmed the general observations and conclusions drawn from the numerical model analyses.

As with all numerical models, the results can be useful for understanding likely mechanisms of failure and for investigating potential solutions. However, the model results should be verified by careful observations and measurements in the field.

Acknowledgments

The authors wish to thank the management and staff at the two participating mines for their willingness to share their experiences and provide data for these analyses. The contribution of Dr. Steve Tadolini of Orica in initiating the analyses at Mine A is gratefully acknowledged.

References

1. Esterhuizen, GS.; Ellenberger, JL. A procedure for the rapid assessment of coal mine roof stability against large roof falls. Proceedings of the 34th international conference on ground control in mining; Morgantown: West Virginia University; 2015.
2. Zhang, P.; Wade, S.; Zeglen, E.; Peterson, S.; Lawrence, R.; Mishra, M., et al. Roof bolting in low seam mining with thick “stack rock” roof under high horizontal stress. Proceedings of the 29th

international conference on ground control in mining; Morgantown: West Virginia University; 2008. p. 11

3. Molinda, GM.; Mark, C. Coal mine roof rating (CMRR): a practical rock mass rating for coal mines. Pittsburgh, Pennsylvania: US Department of the Interior, Bureau of Mines IC 9387; 1994. p. 83
4. Mark, C.; Gadde, M. Global trends in coal mine horizontal stress measurements. Proceedings of the 27th international conference on ground control in mining; Morgantown: West Virginia University; 2008. p. 319-31.
5. Dolinar, D. Variation of horizontal stresses and strains in mines in bedded deposits in the eastern and mid-western United States. Proceedings of the 22nd international conference on ground control in mining; Morgantown: West Virginia University; 2003. p. 178-85.
6. Esterhuizen, GS. A stability factor for supported mine entries based on numerical model analysis. Proceedings of the 31st international conference on ground control in mining; Morgantown: West Virginia University; 2012. p. 25
7. Deng Y, Tang J, Zhu X, Fu Y, Dai Z. Analysis and application in controlling surrounding rock of support reinforced roadway in gob-side entry with fully mechanized mining. Min Sci Technol. 2010; 20(6):839–45.
8. Meng C, Li X, Yao O, Zhou J. Numerical research on stability control of roofs of water-rich roadway. Int J Min Sci Technol. 2014; 24(3):409–16.
9. Esterhuizen, GS.; Bajpayee, TS.; Murphy, MM.; Ellenberger, JL. Validation of entry stability factors determined by the strength reduction method against empirical methods. Proceedings of the 32nd international conference on ground control in mining; Morgantown: West Virginia University; 2013. p. 82-9.
10. Tulu, IB.; Esterhuizen, GS.; Heasley, K. Calibration of FLAC3D to simulate the shear resistance of fully grouted rock bolts. Proceedings of 46th US rock mechanics/geomechanics symposium; Chicago, USA. 2012. p. 12-167.
11. Tadolini, S. Personal communications. Pittsburgh, Pennsylvania: 2014.

Fig. 1.
Laminated silty shale roof rocks observed at Mine B.

Fig. 2.
Large roof fall at Mine A showing laminated nature of the silty shale roof rocks and steep-sided collapse cavity.

Author Manuscript
Author Manuscript
Author Manuscript

Fig. 3.
Severe cutter formation and roof cantilevering at Mine B.

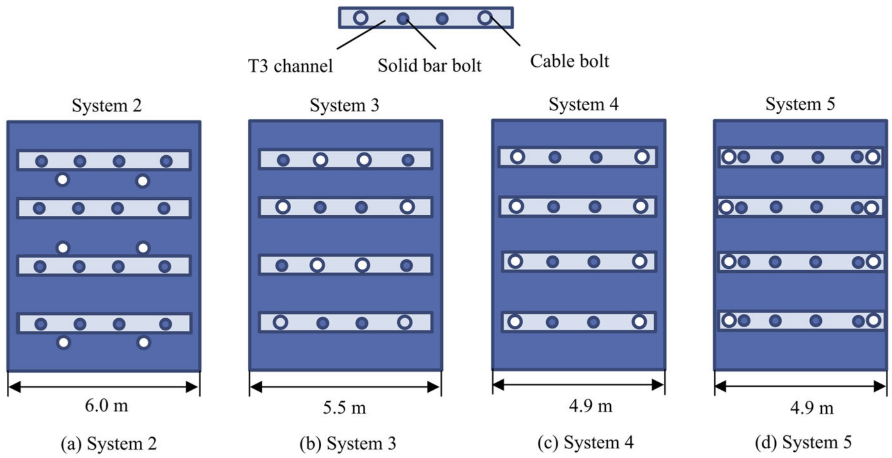

Fig. 4.
Schematic illustration of the support systems evaluated, showing locations of solid bar bolts, cable bolts, T3 channel, and entry widths.

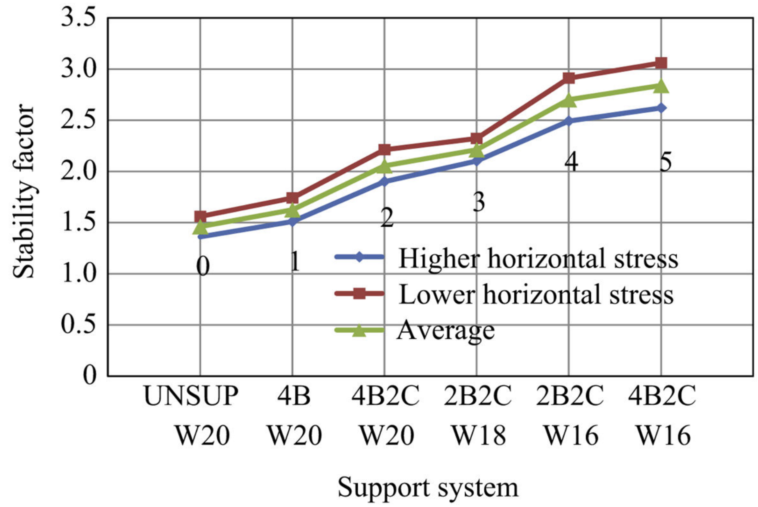

Fig. 5.

Assessment of support systems using a recently developed equation for the prediction of entry stability against large roof falls [1].

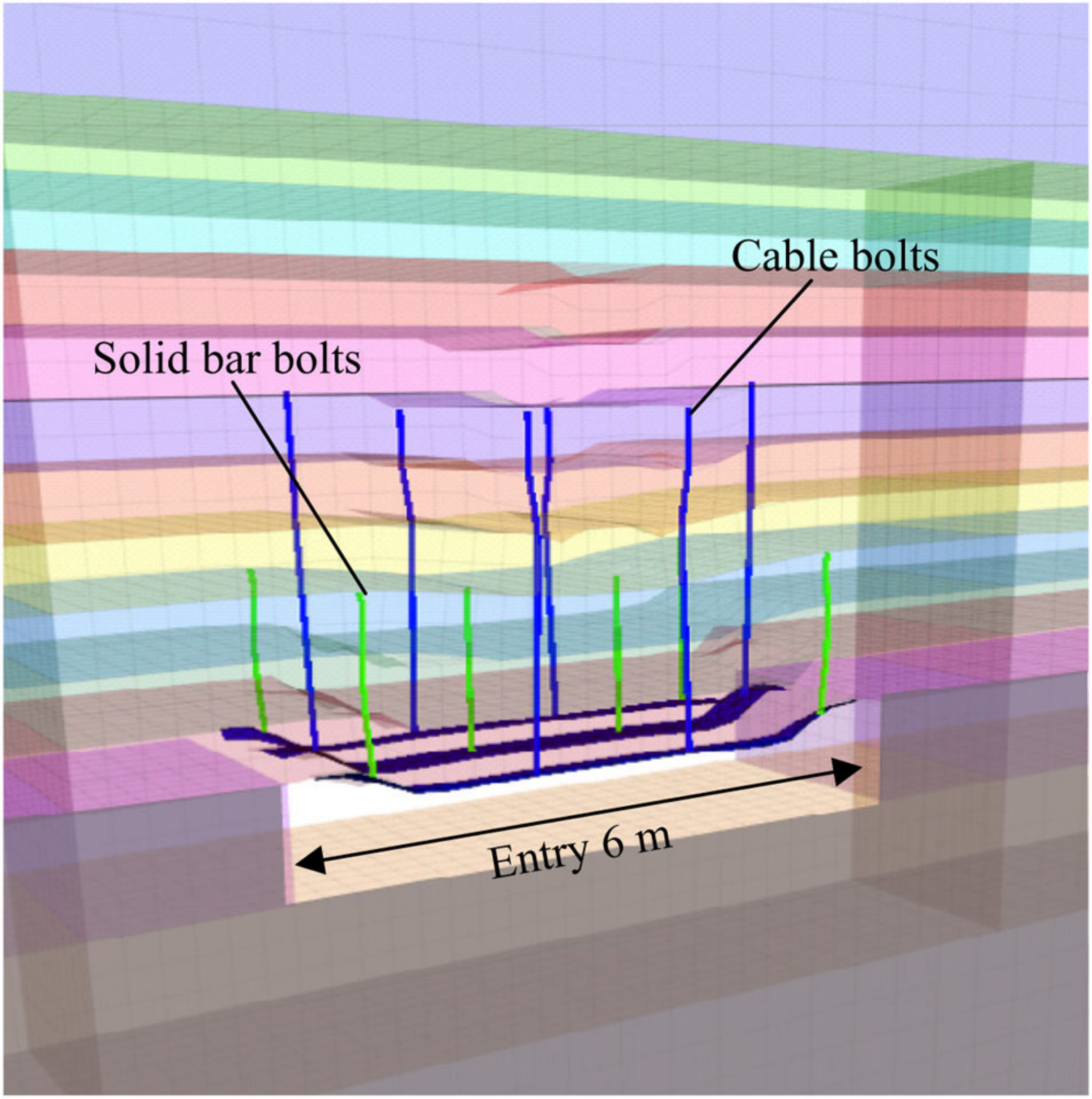

Fig. 6.
Example showing the numerical model used to evaluate support system 3.

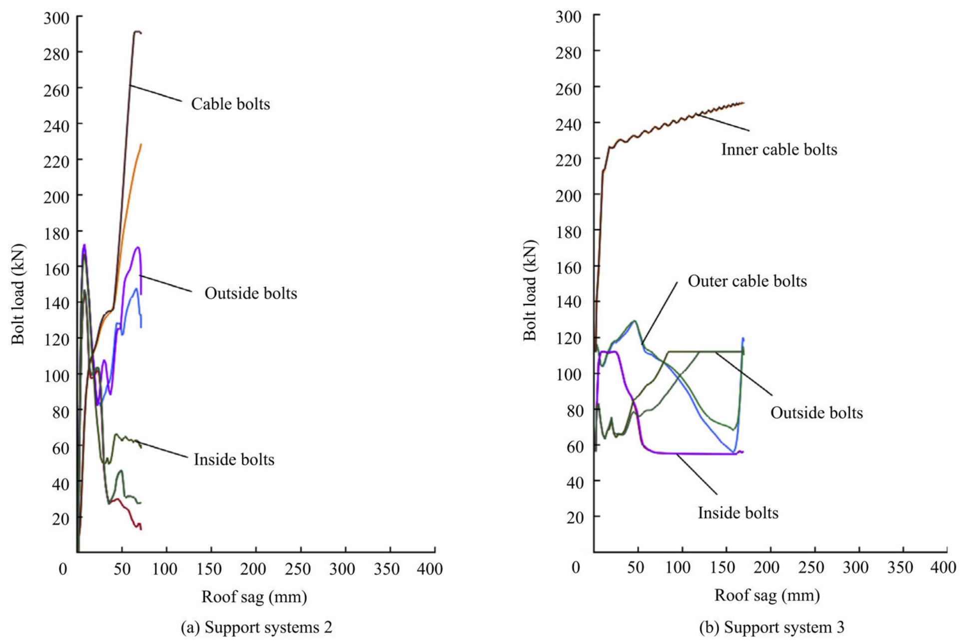

Fig. 7.

Numerical model results showing bolt loads and roof sag for two support systems at the point of critical stability.

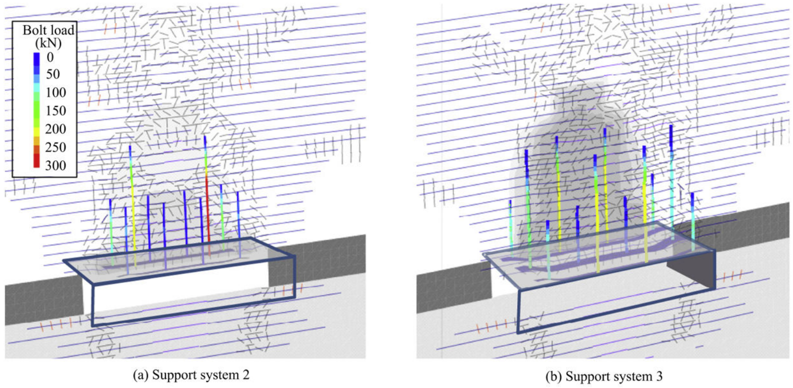

Fig. 8.

Diagrams showing support loads and rock damage around entries for support systems at the point of critical stability, just prior to roof collapse.

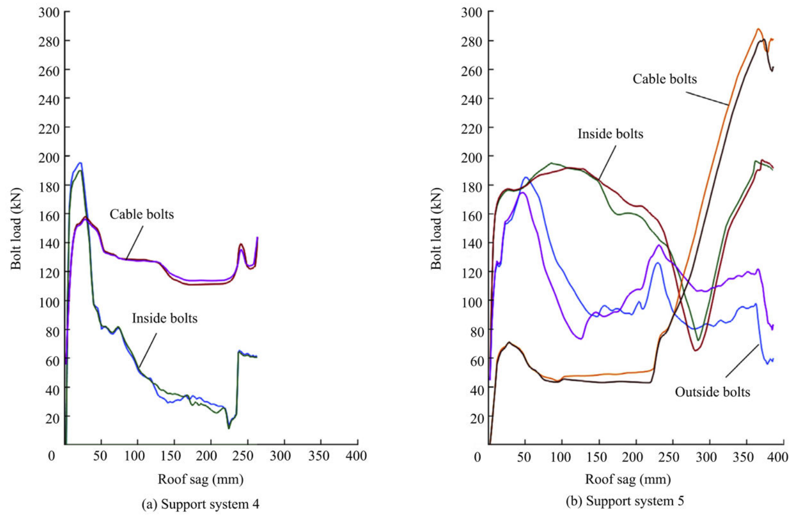

Fig. 9.

Numerical model results showing bolt loads and roof sag for two support systems at the point of critical stability.

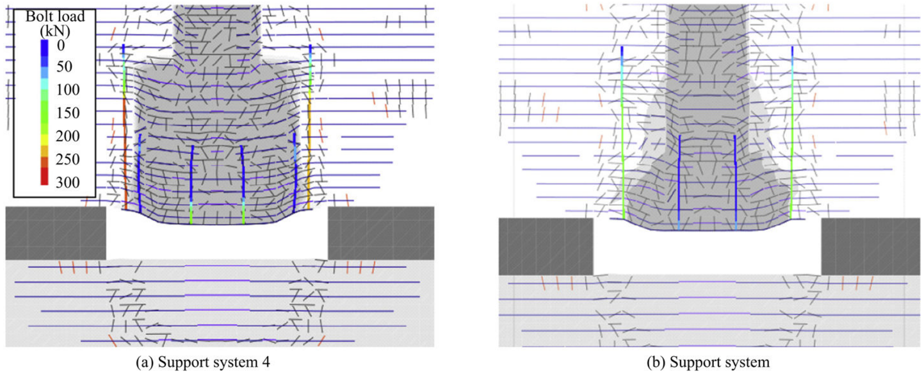

Fig. 10.

Diagrams showing support loads and rock damage around entries for both systems at the point of critical stability, just prior to roof collapse.

Fig. 11.
Example of support system 5 at Mine B.

Author Manuscript
Author Manuscript
Author Manuscript

Table 1

Parameters used for modeling the silty shale roof rocks.

Laboratory-scale UCS (MPa)	Rock mass cohesion 1 (MPa)	Rock mass cohesion 2 (MPa)	Rock mass friction angle (°)	Bedding cohesion (MPa)	Bedding friction angle (°)	Young’s modulus (GPa)	Poisson’s ratio
60.08700	152010.4	7275.0	28	1360.9	23	14.7	0.25